

Iowa ABC Connections



**Final Report
June 2015**



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The preparation of this report was financed in part through funds provided by the Iowa Department of Transportation through its "Second Revised Agreement for the Management of Research Conducted by Iowa State University for the Iowa Department of Transportation" and its amendments.

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Technical Report Documentation Page

1. Report No. InTrans Project 13-462	2. Government Accession No. 	3. Recipient's Catalog No. 	
4. Title and Subtitle Iowa ABC Connections		5. Report Date June 2015	
		6. Performing Organization Code 	
7. Author(s) Justin Dahlberg and Brent Phares		8. Performing Organization Report No. InTrans Project 13-462	
9. Performing Organization Name and Address Bridge Engineering Center Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010-8664		10. Work Unit No. (TRAIIS) 	
		11. Contract or Grant No. 	
12. Sponsoring Organization Name and Address Iowa Department of Transportation 800 Lincoln Way Ames, IA 50010		13. Type of Report and Period Covered Final Report	
		14. Sponsoring Agency Code SPR RB06-013	
15. Supplementary Notes Visit www.intrans.iastate.edu for color pdfs of this and other research reports.			
16. Abstract <p>For several years the Iowa Department of Transportation (DOT), Iowa State University, the Federal Highway Administration, and several Iowa counties have been working to develop accelerated bridge construction (ABC) concepts, details, and processes. Throughout this development, much has been learned and has resulted in Iowa being viewed as a national leader in the area of ABC. However, at this time, the Office of Bridges and Structures does not have a complete set of working standards nor design examples to accompany ABC portions of the bridge design manual (now called the Load and Resistance Factor Design/LRFD Bridge Design Manual).</p> <p>During the fall of 2013, the Iowa DOT constructed a bridge on IA 92 in Cass County using an ABC technique known as slide-in bridge construction. During the design of the Cass County Bridge, several questions were raised about the performance of critical design and construction details: the pile-to-pile cap connection and the polytetrafluoroethylene (PTFE) coated bearing pads on which the bridge would slide.</p> <p>The timing of this specific need and the initiation of this project offered a unique opportunity to provide significant short- and long-term value to the Office of Bridges and Structures. Several full-scale laboratory tests, which included several variations of the pile-to-pile cap connection and bearing pad slides, were completed. These tests proved that the connection was capable of achieving the desired capacity and that the expected coefficient of friction of the bearing pads was reasonably low.</p> <p>Finally, a design tool was developed for the Office of Bridges and Structures to be used on future projects that might benefit from a precast pile cap.</p>			
17. Key Words accelerated bridge construction—ABC—slide-in bridge—friction coefficients—pile cap connections		18. Distribution Statement No restrictions.	
19. Security Classification (of this report) Unclassified.	20. Security Classification (of this page) Unclassified.	21. No. of Pages 41	22. Price NA

IOWA ABC CONNECTIONS

Final Report
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Principal Investigator

Brent Phares
Director

Bridge Engineering Center, Iowa State University

Co-Principal Investigator

Justin Dahlberg
Research Engineer

Bridge Engineering Center, Iowa State University

Authors

Justin Dahlberg and Brent Phares

Sponsored by
the Federal Highway Administration and
the Iowa Department of Transportation
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Preparation of this report was financed in part
through funds provided by the Iowa Department of Transportation
through its Research Management Agreement with the
Institute for Transportation

A report from
Bridge Engineering Center
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-8103
Fax: 515-294-0467
www.intrans.iastate.edu

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ACKNOWLEDGMENTS

The authors would like to thank the Iowa Department of Transportation (DOT) for providing funding and the Federal Highway Administration for state planning and research funds, as well as the Iowa DOT Office of Bridges and Structures for technical support on this project. The authors would like to thank Jim Nelson from the Office of Bridges and Structures in particular for his significant contributions to this research project.

EXECUTIVE SUMMARY

For several years the Iowa Department of Transportation (DOT), Iowa State University, the Federal Highway Administration, and several Iowa counties have been working to develop accelerated bridge construction (ABC) concepts, details, and processes. Throughout this development, much has been learned and has resulted in Iowa being viewed as a national leader in the area of ABC. However, at this time, the Office of Bridges and Structures does not have a complete set of working standards nor design examples to accompany ABC portions of the Iowa DOT bridge design manual (now called the Load and Resistance Factor Design/LRFD Bridge Design Manual).

During the fall of 2013, the Iowa DOT constructed a bridge on IA 92 in Cass County using an ABC technique known as slide-in bridge construction. During the design of the Cass County Bridge, several questions were raised about the performance of critical design and construction details: the pile-to-pile cap connection and the polytetrafluoroethylene (PTFE) coated bearing pads on which the bridge would slide.

The timing of this specific need and the initiation of this project offered a unique opportunity to provide significant short- and long-term value to the Office of Bridges and Structures. Several full-scale laboratory tests, which included several variations of the pile-to-pile cap connection and bearing pad slides, were completed. These tests proved that the connection was capable of achieving the desired capacity and that the expected coefficient of friction of the bearing pads was reasonably low.

Finally, a design tool was developed for the Office of Bridges and Structures to be used on future projects that might benefit from a precast pile cap.

GENERAL INFORMATION

Introduction

For several years the Iowa Department of Transportation (DOT), Iowa State University (ISU), the Federal Highway Administration (FHWA), and several Iowa counties have been working to develop accelerated bridge construction (ABC) concepts, details, and processes. Throughout this development, much has been learned and has resulted in Iowa being viewed as a national leader in the area of ABC.

In fact, the Iowa DOT developed and adopted a departmental policy in 2012 regarding the use of ABC, and, at the same time, the Office of Bridges and Structures began the development of an ABC chapter for its bridge design manual (now called the Load and Resistance Factor Design/LRFD Bridge Design Manual). However, the Office of Bridges and Structures does not have a complete set of working standards nor design examples to accompany the ABC portions of the manual at this time.

During the fall of 2013, the Iowa DOT constructed a bridge on IA 92 in Cass County using an ABC technique known as slide-in bridge construction. During the design of the Cass County Bridge several questions were raised about the performance of critical design and construction details. One specific question was with regard to the ultimate capacity of the pile-to-pile cap connection. The timing of this specific need and the initiation of this project offered a unique opportunity to provide significant short- and long-term value to the Office of Bridges and Structures.

The connection is constructed by first placing a precast pile cap, complete with pile pockets formed using corrugated metal pipe (CMP), over the top of driven steel H-piles at their final location. Then the connection is completed by placing a high-slump structural concrete chip mix into the pile pockets until full, which envelops the pile and creates a positive connection with the CMP.

In addition to being used at Cass County, this connection has been used previously in multiple ABC projects, though all previous designs were smaller in scale and based on a limited number of experimental tests. Testing of a similar connection was completed at ISU previously using HP10x57 piles embedded in a 21 in. diameter CMP. In comparison, HP 14x117 piles and 27 in. diameter CMP were used at the Cass County Bridge and in this investigation. To allow for expanded use of the detail, further examination was needed along with the development of working standards and design examples.

Slide-in bridge construction requires a method that will allow the constructed superstructure to be slid from a temporary position to the permanent position onto a newly constructed substructure. The method prescribed for the Cass County Bridge uses stainless steel sliding shoes on polytetrafluoroethylene (PTFE) topped neoprene bearing pads. The force required to slide the bridge was unknown because the coefficient of friction between the two materials was also

unknown. A test was completed at ISU as part of this project to determine the coefficient of friction between the materials in a lubricated and non-lubricated condition. From these tests, the force required to move the bridge and thus the jacks required can be determined.

Massena, Iowa, ABC Slide-In Project

The Iowa DOT completed its first slide-in project during the fall of 2013. The bridge on IA 92 crosses a small stream immediately west of Massena, Iowa, in southwest Iowa.

Several considerations were made for why accelerated bridge construction was a good solution. First, to close the bridge would have meant a 13 mile detour and 7 mile out of distance travel for road users. This is even more significant when coupled with the fact that 16% of all vehicles were trucks. It is estimated that to complete the bridge replacement using traditional construction methods a road closure of approximately 180 days would have been necessary, resulting in indirect costs of \$437,000 and direct costs of \$15,000.

The Iowa DOT in its pursuit of accelerated bridge construction methods planned to implement slide-in construction methods at a site where it made sense to do so. At Massena, slide-in bridge construction was a good solution for accelerated bridge construction. The existing right-of-way allowed sufficient space for temporary works adjacent to the bridge without having to acquire temporary right-of-way. Moreover, the geography was relatively flat, thereby simplifying the construction.

The existing bridge, originally constructed in 1930, was a 40 by 30 ft steel I-beam structure with high abutment walls and narrow channel passage, as shown in Figure 1.



Figure 1. Original structure

Over its history, it had been reconstructed, retrofitted, and overlaid on several occasions. Soon before its replacement, it achieved a sufficiency rating of 38.2, which indicated that it is structurally deficient. An artist rendering of the proposed replacement is shown in Figure 2.

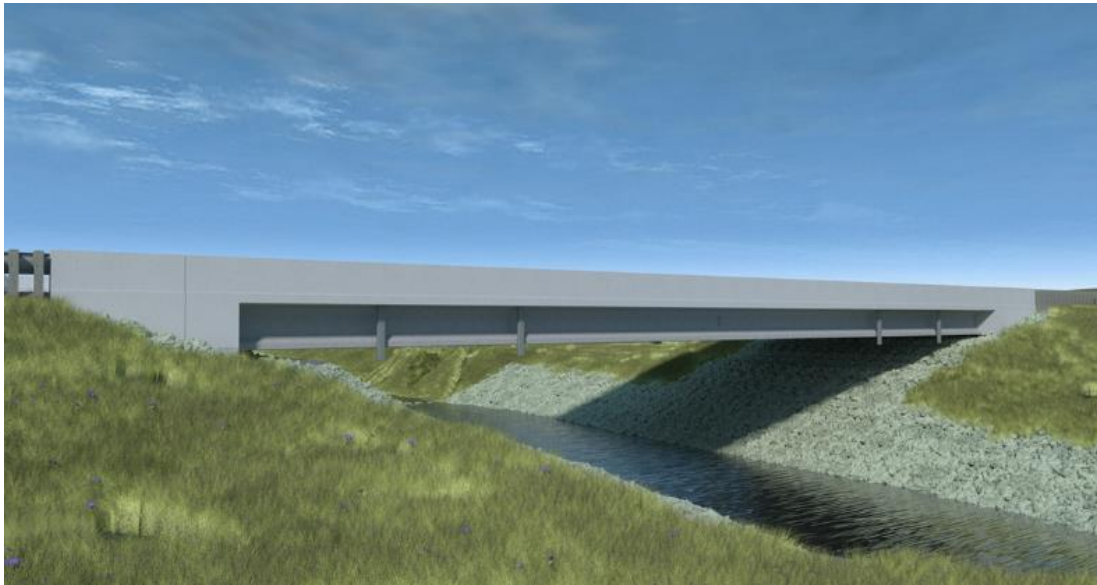


Figure 2. Rendering of proposed replacement

The plan design and details incorporated a semi-integral abutment and abutment diaphragm. The diaphragm served as a block for pushing and pulling the prefabricated structure. Moreover, within the diaphragm, jacking pockets for lifting were formed.

The design called for precast abutment footings set over the top of H-piles, each connected through filling with concrete the corrugated metal pipe void forms within the footing, as shown in Figure 3.

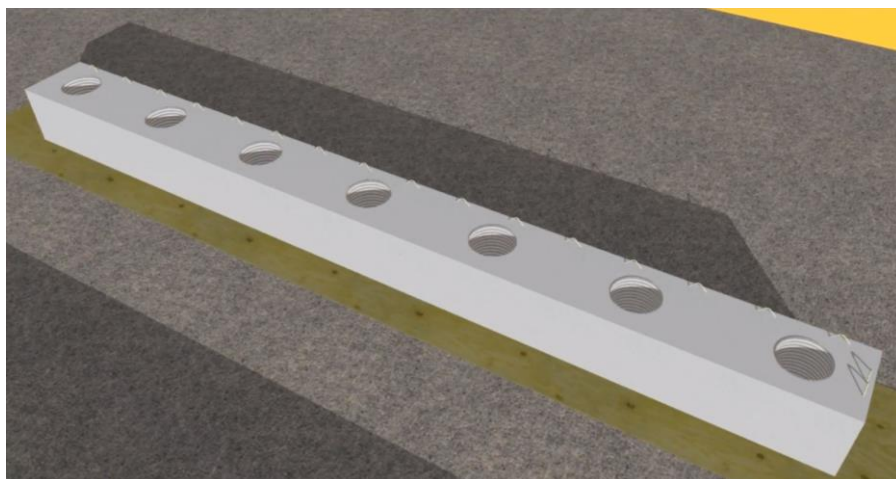
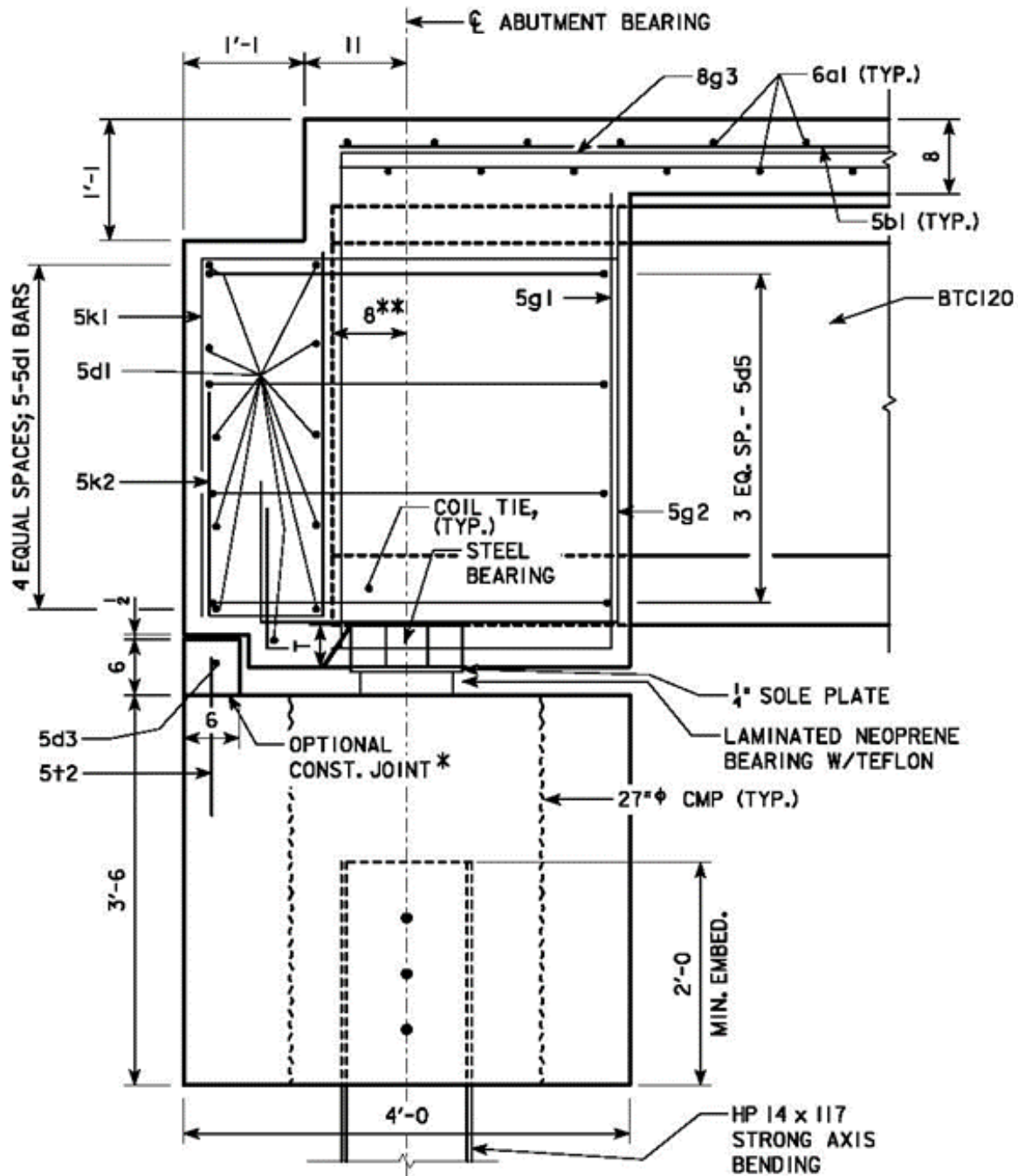


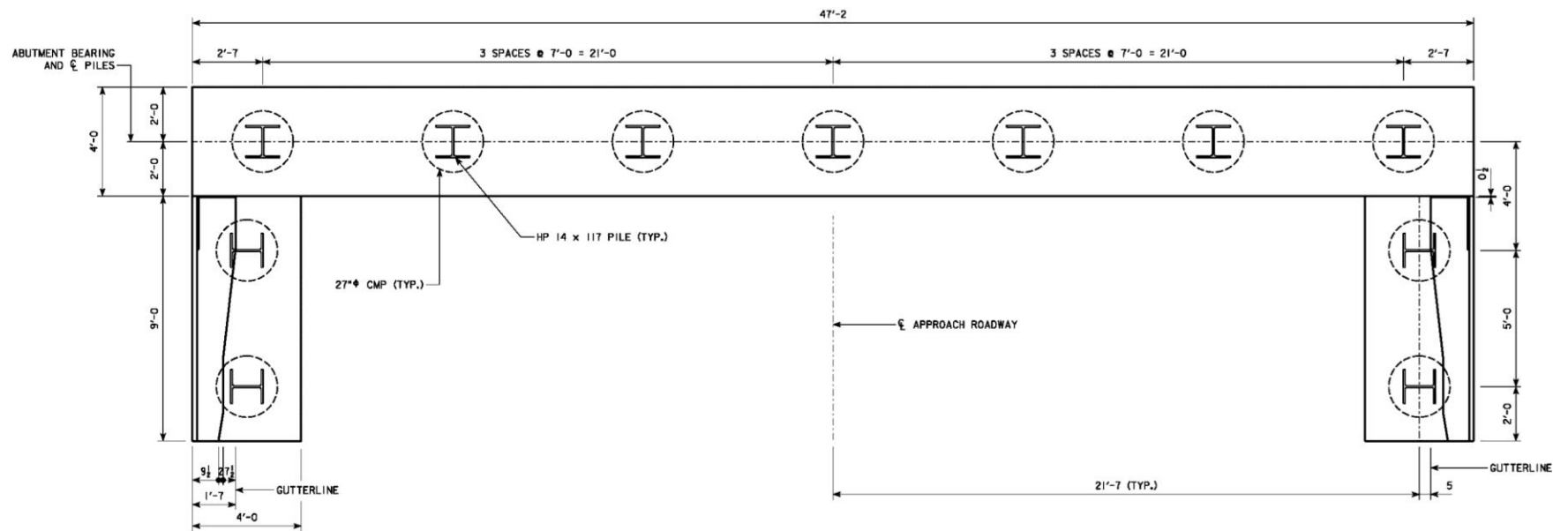
Figure 3. Rendering of pile cap

The H14x117 piles, of which there were seven at each abutment, were spaced at 7 ft 0 in. on center and required 2 ft 0 in. minimum embedment. The corrugated metal pipe measured 2 ft 3 in. in diameter and 3 ft 6 in. in height. Overall, the precast footing measured 4 ft 0 in. wide, 3 ft. 6 in. tall, and 47 ft. 2 in. long. Figure 4 and Figure 5 show the pile and precast abutment footing details.



www.iowadot.gov/MassenaBridge/documents/Plans.pdf

Figure 4. Section through abutment



www.iowadot.gov/MassenaBridge/documents/Plans.pdf

Figure 5. Abutment plan view

Detailed plan notes were provided for the abutment footing construction, and those pertinent to this study are as follows:

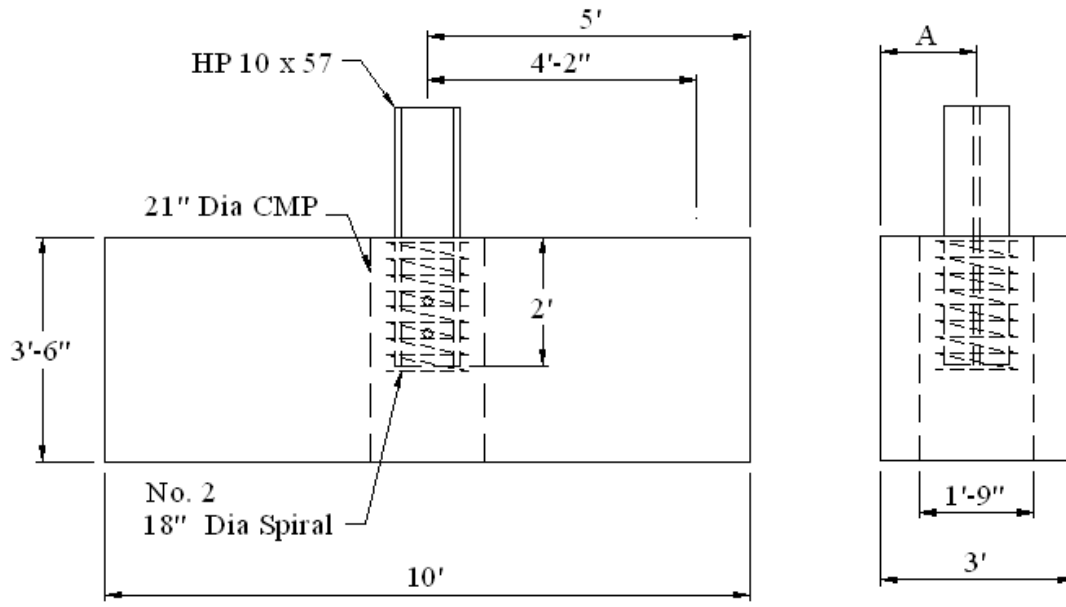
- The required nominal axial bearing resistance for east and west abutment piles is 182 tons at end of drive or retap. The pile contract length shall be driven as per plan unless piles reach refusal. Construction control requires a wave equation analysis of pile driving (WEAP) with bearing graph.
- Final pile head position shall not deviate from the location designated in these plans by more than 3 in. in any direction in order to allow the precast abutment footing and wings to be installed.
- Estimated weight of one precast abutment footing with keeper block is 42.2 tons.
- The structural concrete used to fill the abutment piling encasements shall be class D concrete with a high-range water reducer (HRWR). The maximum slump achieved with water shall be 2 in. The HRWR shall be added at the pour site. The maximum allowable slump after addition of the HRWR shall be 7 in. Course aggregate shall be 1/2 in. top size.
- Precast footing and wingwall concrete, $f'_c = 5,000$ psi.

Literature Review

Push-Through Tests

Wipf et al. (2009) completed an investigation of pile-to-pile cap connections using corrugated metal pipe void forms in pile caps. The study provided proof testing for a connection intended for a bridge to be constructed in Boone County, Iowa. In the study, eight laboratory tests were completed consisting of five single abutment tests, two double pile abutment tests, and one pier cap test. Of all the tests completed, the five single abutment tests were most similar to the tests conducted for this study and are discussed in the following paragraphs.

The connections were designed by the Iowa DOT to support a service load of 80 kips. Within the single pile test specimens, HP 10x57 piles and 21 in. diameter CMP were used (see Figure 6).



Wipf et al. 2009

Figure 6. Single pile abutment test specimen

Each of the pile cap specimens measured 36 in. wide, 42 in. deep, and 10 ft long and were supported at the spacing (8 ft 4 in.), which approximately corresponded with the beam spacing. The piles were embedded 24 in. into the CMP void form, and the shear connectors used to connect the pile and 6,000 psi compressive strength CMP infill were two 7/8 in. diameter studs on each side of the pile web spaced at 6 in. on center and 6 in. from the end of the pile. In addition, no. 2 18 in. diameter spiral reinforcement was placed into the CMP surrounding the pile specimen.

The specimens were tested using hydraulic jacks and load frames by simply applying axial load to the pile and recording pertinent strain and deflection data. Each of the specimens was loaded to the maximum 400 kip capacity of the actuator, over four times the unfactored design load, without failing and without the researchers seeing any signs of imminent failure.

Punching Shear

Ospina and Hawkins (2013) discuss the current state of practice with regards to punching shear and the origins of that practice. For about 50 years, the American Concrete Association (ACI) and the American Association of State Highway and Transportation Officials (AASHTO) have used the equation $V_n = 4\sqrt{f'_c}b_o d$ (where f'_c is the specified compressive strength of concrete, b_o is the critical perimeter measured at $0.5d$ from the column face, and d is the effective depth of the flexural reinforcement in the slab), developed by Moe in 1961. This equation was developed and primarily intended for calculating the punching shear capacity of two-way reinforced concrete (RC) slabs. Though the equation has largely served the civil industry well for nearly a half-century, some question its validity with the introduction of higher strength steels and concretes. Moreover, the equation is commonly used in applications such as footings where the

thickness and span lengths are very unlike a two-way RC slab. These concerns aside, the authors provide a good argument for the continued use of the equation as a design equation rather than a shear capacity predictor, which many think it is.

Hallgren et al. (1998) performed numerous small-scale tests to identify the effects of concrete strength, ratio of flexural reinforcement, reinforcement anchorage, and type of shear reinforcement on punching shear strength in column footings. The tests indicated that, of all the variables, concrete strength had the most influence on punching shear strength, while the other variables had no influence or only slight influence. Of particular interest were the behavior changes in punching shear for specimens with low shear-span ratios, i.e., column footings. The angle of the punching shear crack observed in the tests of the column footings was between 50 and 60 degrees, which is much steeper than the shear-crack slopes observed in previous tests on more slender slabs. With this change in geometry of the failure plane and the significant influence of concrete strength, modifications to the well-known punching shear design equations were proposed, including a multiplier for concrete strengths above or below 5.07 ksi.

LABORATORY TESTS

Pile Cap Tests

The primary objectives of the laboratory tests were to identify the push-through capacity of the pile and CMP assemblies and to assess the differences in performance when variations of shear transfer mechanisms between the pile and concrete-filled CMP were used.

Nine total specimens of the centermost portion of the abutment footing were constructed at full scale; measuring 11 ft 0 in. long, they encompassed one pile and its respective CMP void form. See Figure 7 and Figure 8.

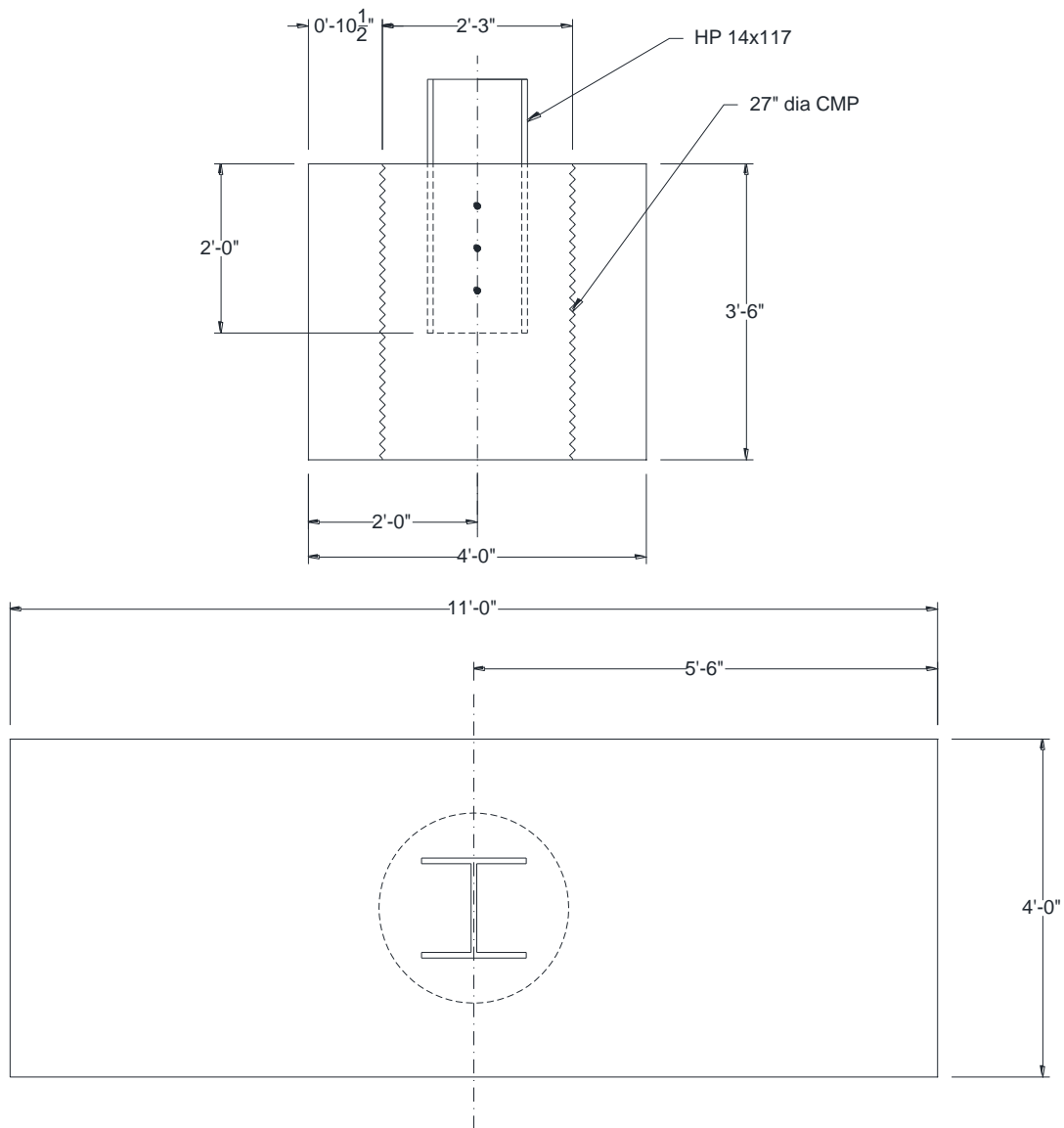


Figure 8. Laboratory specimen plans

Other than the overall length being reduced, all other aspects of the specimens exactly followed the plan details (e.g., reinforcement bar sizes, placement, and coatings). Figure 9 and Figure 10 show a couple of the specimens under construction.



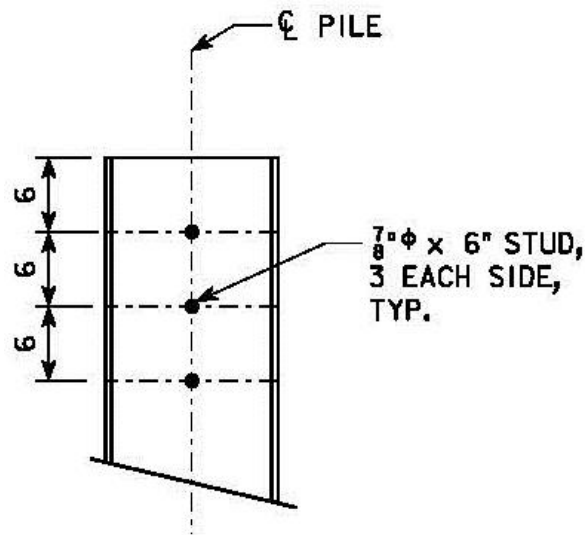
Figure 9. Laboratory specimen construction



Figure 10. Completed laboratory specimen

The structural concrete used for the precast pile cap was a standard Iowa DOT C-4, 5,000 psi mix. The CMP elements were 27 in. in diameter and constructed of 16 gage galvanized corrugated steel, Type I. The steel pile size used was HP 14x117. The structural concrete used to fill the CMP void forms was a class D concrete with a high range water reducer added on site and an aggregate size no greater than 1/2 in.

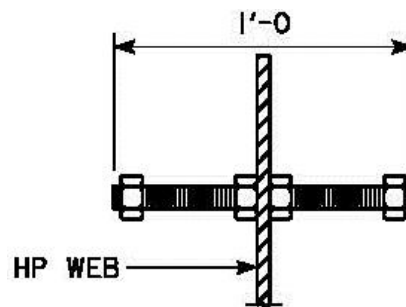
Three variations of shear transfer mechanisms were evaluated; three specimens for each variation were constructed. The first, shown in Figure 11, required three 7/8 in. diameter by 6 in. long shear studs spaced at 6 in. to be shot to each side of the pile web on centerline.



www.iowadot.gov/MassenaBridge/documents/Plans.pdf

Figure 11. Shear stud layout

The second, shown in Figure 12, replaces the shear studs with three 7/8 in. diameter by 1 ft 0 in. long threaded F1554 Grade 36 anchor rods with four A563 Grade A hex nuts.



www.iowadot.gov/MassenaBridge/documents/Plans.pdf

Figure 12. Alternative shear stud assembly

Holes were to be drilled or punched in the same locations as the studs. The last variation was to remove shear connectors altogether. An example of each assembly is shown in Figure 13, Figure 14, and Figure 15, respectively.



Figure 13. Shear connector - shear studs



Figure 14. Shear connector - threaded rods



Figure 15. Shear connector - no connectors

Once completed and the concrete allowed to cure to specified strength, the specimens were tested for push-through strength. For simplicity, the specimens were constructed and tested upside down, as shown in Figure 16; the specimens were elevated and supported at locations corresponding to the beam seats and loaded from above.



Figure 16. Test specimen in reaction frame

Four 120 kip actuators were ganged for a total load capacity of 480 kips and placed between the pile and reaction frame. See Figure 17.



Figure 17. Load actuators

To more accurately assess the performance under load, each of the specimens was instrumented with numerous strain and deflection gages. Four strain gages, one at each pile flange, were placed near the edges, as shown in Figure 18, to quantify any eccentricity that might have resulted during loading.



Figure 18. Instrumentation at top of specimen

An additional four strain gages were placed directly on the abutment footing, one on each side near the top and bottom of the concrete to quantify any possible bending. Any differential deflection between the footing portion and the CMP infill on top of the specimen was measured using four deflection gages at each quadrant of the CMP. The differential deflection at the bottom of the specimen was measured in a similar way, as shown in Figure 19.



Figure 19. Instrumentation at bottom of specimen

Pile Cap Test Results

Each specimen was loaded to the capacity of the load frame and jacks, 480 kips. In every case, the load was insufficient to determine the ultimate capacity of the specimen. Nonetheless, valuable data were collected from which a probable failure mechanism was identified.

Figure 20 and Figure 21 show the differential deflection results measured at the top and bottom of each test specimen, respectively. The magnitudes at the top are quite small, ranging from almost 0.00 in. to the maximum, which is slightly greater than 0.01 in.

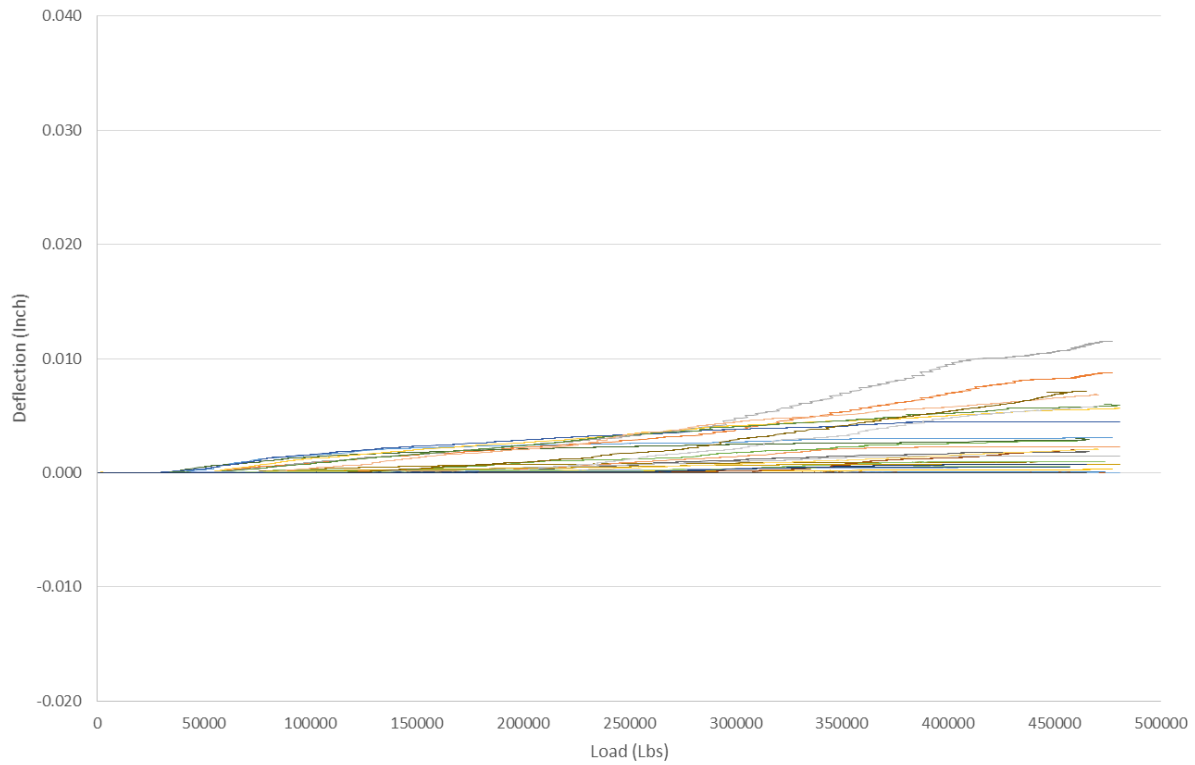


Figure 20. Differential deflection at top of specimen

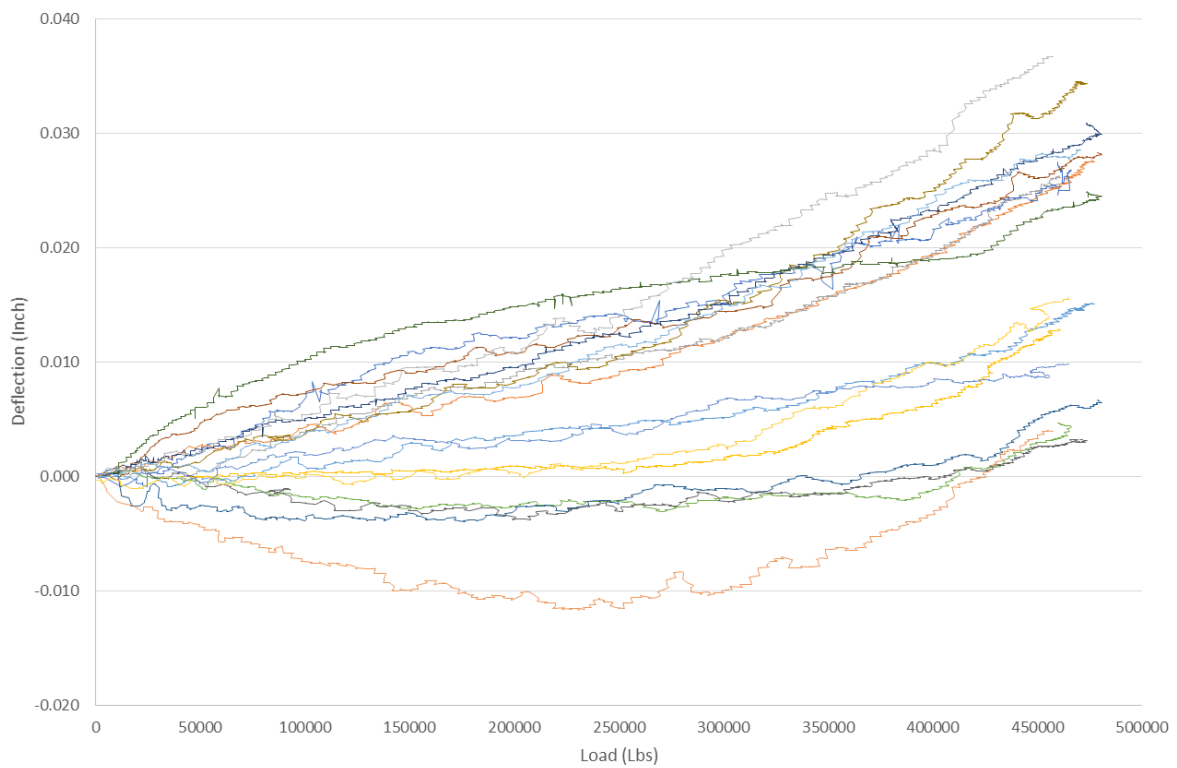


Figure 21. Differential deflection at bottom of specimen

Conversely, the combined differential deflection at the bottom of each specimen, though still quite small, is relatively large in comparison to that at the top of the specimen. A maximum differential deflection was measured to be slightly less than 0.04 in.

Moreover, no appreciable difference was seen between specimens with varying shear transfer mechanisms. Therefore, the researchers believe the connectors to have negligible effects and disregard them in their development of design equations; the collected strain data corroborate this idea.

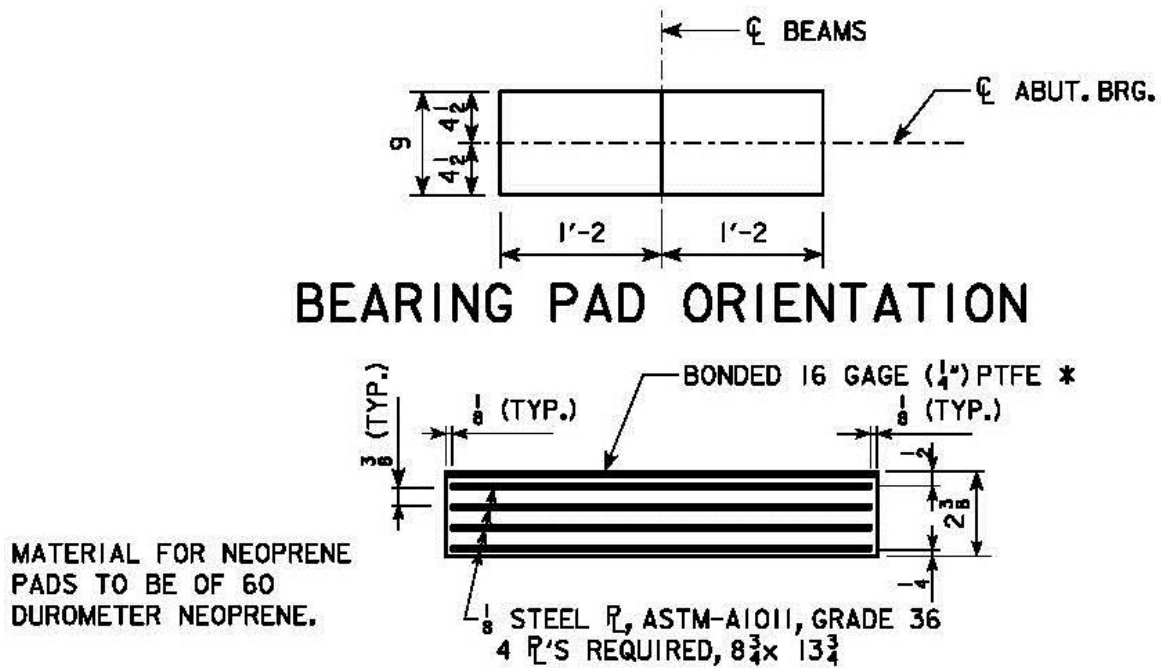
Together, the differential deflection results observed at both the top and bottom of the specimen indicate a discontinuity in overall movement of the void concrete (more movement at the bottom than at the top), and, without the ability to determine the ultimate failure mechanism, the researchers were left to their engineering judgment and interpretation of the measured response. As such, the results would indicate that the void concrete below the end of the pile was engaged to a greater degree than that which was above, and a possible punching shear failure would ultimately occur originating at the end of the pile if loading continued. It is this theory that led to the development of a design methodology and accompanying equations.

Coefficient of Friction Tests

To help determine the force required to slide the new bridge superstructure onto the permanent substructure, a test was developed to determine the coefficient of friction between the sliding shoes and bearing pads.

The static and kinetic coefficient of friction (CoF) between the stainless steel sole plate and PTFE-topped neoprene bearing pads was determined by applying the anticipated service dead load to the assembly and pushing laterally until movement was obtained and sustained. It was by measurement of the loads required for movement that the friction coefficients were calculated. In a similar fashion, the behavior of the bearing pads was observed during testing to determine if excessive shear deformation occurs such that the bearing pads may “roll” during construction.

The PTFE-topped neoprene bearing pads, of which there were two, measured 9 in. wide by 14 in. long by 2 3/8 in. tall, and the PTFE was 16 gage per the plan details. The neoprene bearing was reinforced with four 1/8 in. steel plates embedded within the 60 durometer neoprene. The bearing pad detail is shown in Figure 22.



www.iowadot.gov/MassenaBridge/documents/Plans.pdf

Figure 22. Bearing pad details

The sole plate was constructed and then tested in an inverted position with respect to its as-built position, as shown in Figure 23 and Figure 24.



Figure 23. Coefficient of friction test setup



Figure 24. Inverted bearing pad on stainless steel sole plate

In addition, where in the actual construction the sole plate embedded in the integral abutment would slide over the stationary bearing pads, in the testing scenario the sole plate remained stationary while the bearing pads slid against the sole plate.

The anticipated maximum service dead load of 60 tons was applied to the bearing pad assembly during the slide operation. This loading was simulated in the laboratory by applying a vertical load of the same magnitude using a hydraulic actuator. Once this load was applied, a continuously increasing horizontal load was applied, also using a hydraulic actuator, and recorded until the bearing pads began to slide along the sole plate, at which time the applied load steadied.

Two variations of the test were completed. The first variation consisted of measuring the coefficient of friction between the PTFE-coated bearing pad and the stainless steel sole plate, while the second variation introduced a lubricant (Dawn dish soap) to determine any reduction in the coefficient of friction. Multiple tests were completed for each variation.

During the slide, any deformation of the bearing pads that occurred was observed and recorded.

Coefficient of Friction Test Results

The total anticipated load required to move the bridge superstructure from its temporary offline position to its final position could be quantified through the coefficient of friction tests. Prior to

the tests being completed, only an estimate of the load could be figured based on available information from a limited number of completed bridge slides.

The CoF was calculated by dividing the horizontal (sliding) load by the vertical (dead) load as shown below.

$$\text{Coefficient of Friction} = \frac{\text{Horizontal Load}}{\text{Vertical Load}} \quad (1)$$

By observation, one can see in Figure 25 that there is a noticeable difference between the CoF in the non-lubricated and lubricated tests. Two distinct bands exist that are differentiated by the use of lubricant; each is labeled within Figure 25.

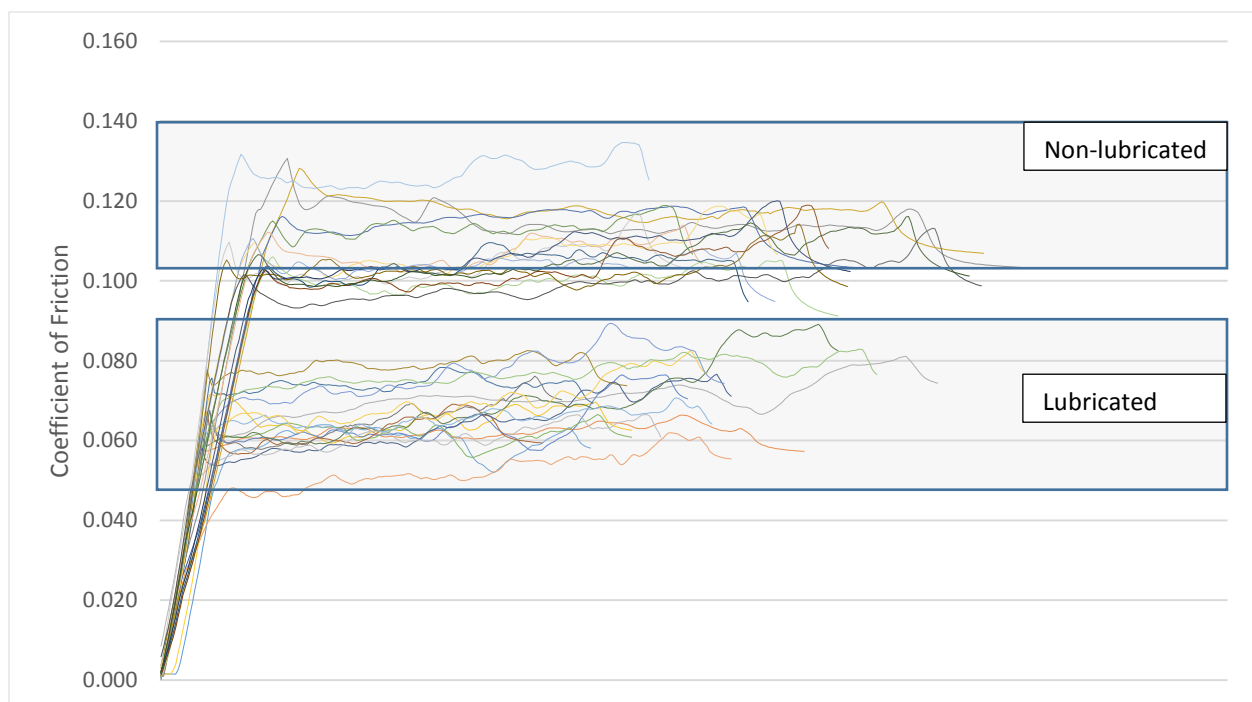


Figure 25. Coefficient of friction test results

It is also noteworthy that the static and kinetic CoF does not greatly differ in each of the variations, though one should note that the vertical loading was completed immediately prior to the horizontal sliding, whereas in actual construction the dead load of the superstructure may be atop the bearing pads for weeks or even months, resulting in a “locked” state. The load required to initiate sliding during construction may be slightly higher than what was needed in the laboratory tests.

On average, for the non-lubricated tests the CoF was calculated to be approximately 0.11, whereas for the lubricated tests the CoF was calculated to be approximately 0.07, a reduction of 36 percent of the load required for sliding, which could lead to a smaller jack being required.

During the tests, a noticeable deformation of the bearing pad occurred. The shear deformation began immediately upon the initiation of the horizontal loading and discontinued once the initial break from static to kinetic state was achieved. Shown in Figure 26 are several lines that help highlight the deformation of the bearing pad; the total deformation was measured to be 10 degrees from vertical.

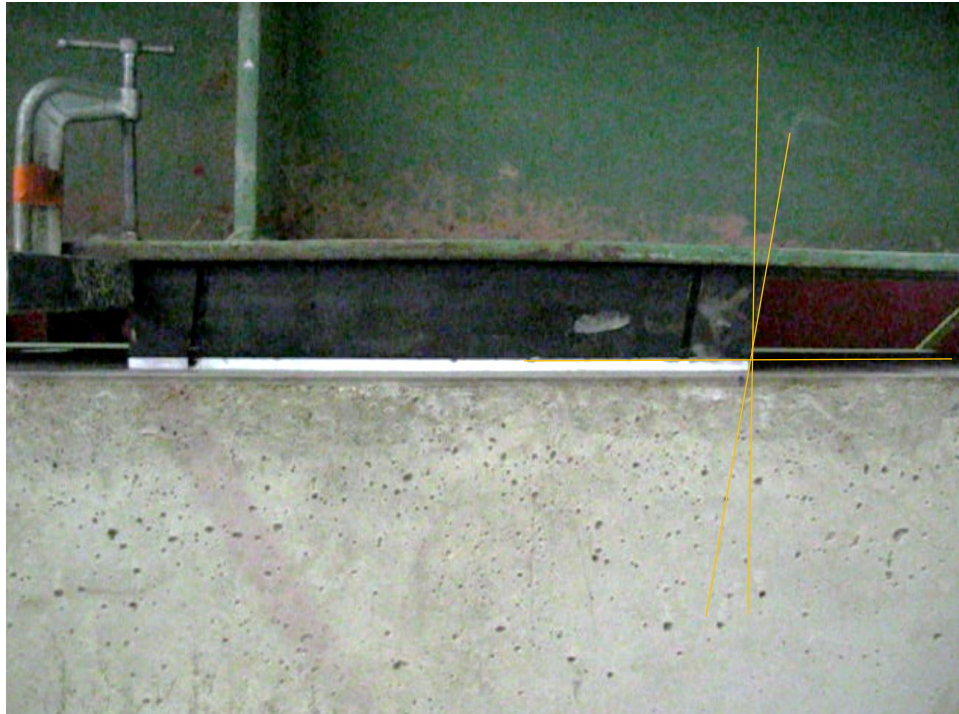


Figure 26. Bearing pad shear deformation

Although the deformation was noticeable, it did not appear to be significant enough to cause the bearing to “roll”. Also, once the test was complete and the vertical load removed, the bearing pads immediately returned to form.

DEVELOPMENT OF DESIGN TOOL

Numerous variables exist when considering the design and constructability of a pile cap with CMP void forms. For example, the diameter of the CMP has to be such that it will fit within the width of the pile cap and allow for adequate reinforcement between the CMP and edge of cap. In addition, the pile size must be such that it is not too large for the CMP.

To limit the potential geometric conflicts, the researchers established several guidelines that must be adhered to before the nominal capacity can even be calculated. They are as follows, as illustrated in Figure 27:

- The clearance between the CMP and side of pile cap must be greater than or equal to 5 1/8 in.
- The clear distance between the inside of the CMP and effective pile diameter is 1/2 in.
- The pile driving tolerance is ± 3 in. in any direction
- The minimum distance from end of pile to top of pile cap is 12 in.
- The minimum embedment of the pile is 12 in. (note that only push-through tests were completed and not pull-out tests)

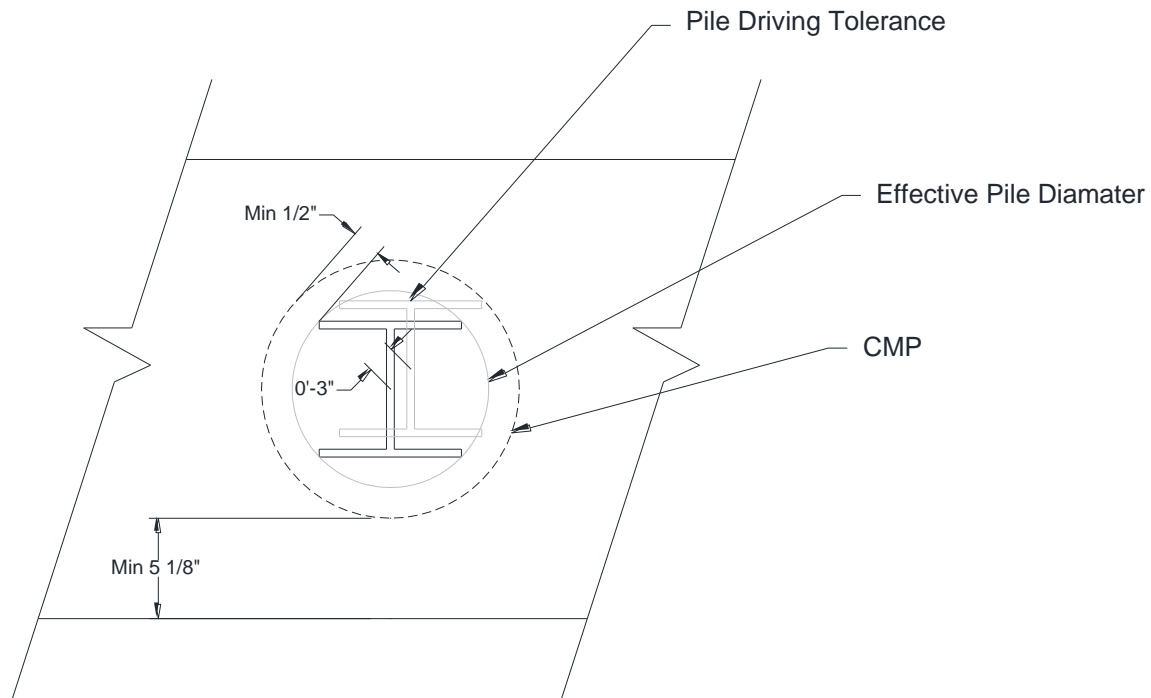


Figure 27. Geometric limitations

The capacity equations are composed of two parts resulting from the two distinct failure planes assumed as follows: failure plane propagating from the pile to the CMP wall and shear friction failure at the CMP and concrete interface. These planes are illustrated in Figure 28.

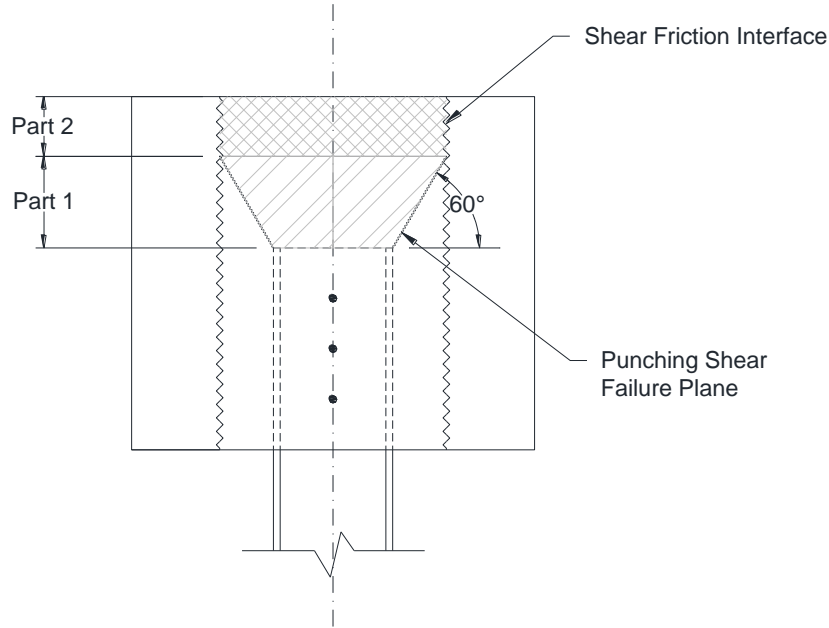


Figure 28. Failure planes

The current AASHTO (2012) design equations for punching shear (5.13.2.5.4-1, 2, and 3) are largely based on the research conducted and published by Moe (1961). Although the equations have stood the test of time for the purposes of design, the equations were not necessarily intended for the purposes of calculating the capacity of deep concrete elements such as footings or, in this case, pile caps; rather, they were intended for two-way elevated reinforced concrete slabs.

More recent research (Hallgren et al. 1998) has aimed to address the influences of such things as flexural reinforcement, shear reinforcement, and concrete strength on deeper sections (column footings). Accordingly, equations were proposed to more accurately calculate the punching shear capacity of deeper sections; the AASHTO equations inherently possess some conservatism. The researchers of this ABC study adopted the equation in Part 1 of the two-part design capacity equation:

$$V_n = \frac{\mu(\pi \times b \times d)}{\left(\frac{5.07}{f'c}\right)^{0.76}} \quad (2)$$

where,

$$\mu = 0.6(f'c)$$

b = effective pile diameter

d = effective depth

$f'c$ = concrete strength, ksi

The second part accounts for the shear friction between the CMP wall and the void concrete interface. AASHTO design equations for shear friction (5.8.4.1-1, 2, 3) calculate the nominal shear resistance of the interface plane. In the absence of reinforcement across the interface and permanent compressive force normal to the shear plane, the equation simply becomes a function of the area in contact and the cohesion factor. In this case, it was believed that the interface between the CMP and void concrete was most analogous to the condition of concrete placed against clean, hardened concrete with surface intentionally roughened to an amplitude of 0.2 in. Thus, Part 2 of the two-part equation becomes the following:

$$V_n = cA_{cv} \quad (3)$$

$$V_n \leq 0.2f'cA_{cv}$$

$$V_n \leq 0.8A_{cv}$$

where,

$$c = 0.100 \text{ ksi}$$

$$A_{cv} = \text{area of concrete engaged in shear transfer (in}^2\text{)}$$

In the end, the total nominal shear resistance is calculated by the following equation subject to the limitations previously stated:

$$V_n = \frac{\mu(\pi \times b \times d)}{\left(\frac{5.07}{f'c}\right)^{0.76}} + cA_{cv} \quad (4)$$

Using this equation, a design tool was produced within which several variables can be modified (e.g., depth of pile cap, width of pile cap, void concrete strength, depth of pile embedment, size of pile). This tool not only provides the ability to calculate the nominal shear capacity of the connection quickly and with a vast number of combinations, but also performs the checks to ensure that the prescribed geometric rules are followed. One should note, however, that the tool is used to provide the calculated design capacity of the CMP to pile connection but does not check the geotechnical capacity of the pile nor design the pile cap. Figure 29 shows an example of the design tool interface, with User Input at the top, Illustration in the middle under that, and Results (showing calculation output) at the bottom.

User Input

Pile Size

HP 12x53

HP 14x57

HP 10x42

CMP Diameter

30

33

36

in

Cap Width

45

48

51

in

Cap Depth

▲

▼

43

in

Min CMP Cover

▲

▼

6.125

in

Pile Embedment

▲

▼

16

in

Concrete Strength

▲

▼

3000

psi

Illustration

Results

Geometry Checks

CMP Size OK

CMP Cover OK

Pile Stress

50

ksi

Pile Capacity

752

kips

Shear Capacity

891

kips

Design Capacity

752

kips

Pile Controls

Figure 29. Design tool interface

SUMMARY AND CONCLUSIONS

Summary

The planned use of accelerated bridge construction methods by the Iowa DOT in 2013 to construct a bridge in Massena, Iowa, led to a laboratory investigation of two design details: pile-to-pile cap connections and PTFE-coated neoprene bearing pads for sliding the bridge superstructure.

The pile-to-pile cap connections use precast pile caps fitted with CMP void forms within which the connection between the pile cap and pile is made. In total, nine laboratory tests were completed: three investigating the use of headed stud shear connectors on the pile, three investigating the use of threaded rods with heavy hex nuts passing through the pile web, and three investigating the use of no shear connectors. The required strength of the connection was validated through testing for each shear connector type, and a probable method of ultimate failure was determined in the event of extreme overloading; none of the specimens could be completely failed due to the capacity limitations of the load frame and actuators.

In addition, using the results from the laboratory testing, a design tool was developed to enable engineers to quickly assess the connection strength of various combinations of piles, CMPs, pile cap sizes, concrete strengths, etc. Working standards and design examples are being developed within the Office of Bridges and Structures to accompany the ABC portions of the Iowa DOT bridge design manual (now called the Load and Resistance Factor Design/LRFD Bridge Design Manual).

The PTFE-coated bearing pads in conjunction with stainless steel slide shoes were investigated for durability under slide conditions and to determine the expected static and kinetic coefficient of friction with and without the use of a lubricant. At anticipated service loading levels, the bearing pads performed well under lubricated and non-lubricated conditions and did not appear to be at risk for “rollover” during the slide. The static and kinetic coefficient of friction were calculated to be approximately 0.11 for the non-lubricated tests and 0.07 for the lubricated tests.

Conclusions

The following conclusions were made as a result of this study:

- The pile-to-pile cap connection does not require a shear connector between the pile and CMP infill.
- The pile-to-pile cap connection greatly exceeds the strength required to withstand the anticipated service loads of the Massena Bridge.
- The coefficient of friction between the stainless steel slide shoe and PTFE-coated neoprene bearing pad is approximately 0.11 for non-lubricated conditions and 0.07 for lubricated conditions, a 36 percent reduction in the required jacking force.

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